Chapter 10
Standards and Specifications for Deformation Monitoring Reference Networks

10-1. General Scope

This chapter details USACE standards and specifications for establishing external deformation monitoring reference networks, from which periodic PICES observations are made to structure object/target points. Also included are requirements and procedures for periodically monitoring the stability of the reference network points. The standards and procedures for establishing reference networks are intended to support the Reference Line Ratio (RLR) technique, also known as the "Robertson Method," of periodic structural monitoring. The RLR/Robertson technique is recommended for most USACE PICES monitoring applications and is further described in Chapter 11.

a. Accuracy standards and specifications. The accuracy standards developed in this chapter apply to deformation surveys carried out by conventional, photogrammetric, and GPS survey methods. It is possible that two or even three of these methods can be used together; therefore, the standards are based upon the magnitude of a detectable single point movement and are independent of the method used. Alternately, the specifications sections are treated as a function of the procedures required, rather than by method used. Both the standards and specifications developed are adapted from a consolidation of information from various USACE commands and other deformation monitoring agencies.

b. Geodetic survey methods. The standards and specifications are based on the determination of deformations using traditional geodetic survey techniques and equipment, as observed from reference networks. The traditional geodetic survey accuracy "orders" (i.e., First-Order, Second-Order) do not apply to deformation surveys since most horizontal and vertical observations are of relatively short length (less than 1,000 m). Generally, traditional geodetic First-Order techniques (not accuracy standards) are followed in PICES surveys.

10-2. Reference Network Accuracy Standards

Table 10-1 depicts the accuracy required in a reference network to obtain the target accuracies outlined in Chapter 9.

Table 10-1
Reference Network Relative Accuracy for Concrete and Earthen Embankment Structures

Linear Extent of Network	Concrete (mm)	Embankment (mm)
Absolute External Observations		
100-1,000 m	5	15
1 km - 5 km	20	50
Relative Deflection Observations (Micrometers)		
10 m	0.5	2
100 m	1	13

Linear extent of deformation network is the maximum spatial distance between the two most widely separated points in the network.

a. From Table 10-1, the relative accuracies of reference points spaced at a typical distance of 1 km are 5 mm for a concrete structure and 15 mm for an embankment structure. These relative accuracies can be readily achieved using careful geodetic survey techniques and instrumentation. This includes use of traditional theodolites and EDM, or full use of modern electronic total stations. Centering errors are minimized through use of forced-centering instrument/target mounts. Use of the RLR/Robertson method minimizes refractive errors in EDM measurement.

b. The relative accuracies of alignment reference points used for short-range deflection observations (usually with micrometers) are as indicated. The use of forced-centering is critical for obtaining repeatability in deflections, and the absolute relative accuracy indicated is only representative should these points be used for external monitoring.

10-3. Accuracy Requirements for Surveying PICES Targets

Table 10-2 depicts the recommended accuracy by which target points on the structure must be located during periodic PICES surveys taken from the reference network. Accuracies indicated are absolute -- relative to the entire reference network, not over a single line (which will be far more precise).

a. Concrete structures. To obtain the indicated 10-mm accuracy, traditional geodetic survey procedures

Table 10-2
Typical Accuracy Requirements for PICES Surveys

Concrete Structures	
Dams, Outlet Works, Locks, Intake Structures	
Long-Term Movement (Geodetic survey methods)	10 mm
Relative Short-Term Deflections Crack/Joint Movements Monolith Alignment (Precision micrometer alignments)	0.01 in. (0.2 mm)
Vertical Stability/Settlement (Precise geodetic leveling)	2 mm
Embankment Structures	
Earth-Rockfill Dams, Levees	
Slope/crest Stability (Total station/DGPS)	0.1 foot
Crest Alignment (Total station/DGPS)	0.1 foot
Settlement measurements (Differential leveling)	0.05 foot
Control Structures	
Spillways, Stilling Basins, Approach/Outlet Channels, Reservoirs	
Scour/Erosion/Silting (Hydrographic surveys)	0.2 to 0.5 foot

are employed. Use of the RLR/Robertson method will easily meet this accuracy standard. Relative accuracies over individual lines will be far more precise. Precise EDM and/or electronic total stations with forced-centering mounts are recommended for performing periodic observations.

b. Embankment structures. Most conventional precise survey methods will provide the accuracies required for these structures. Force-centered electronic total stations are recommended, although tripod-mounted theodolites/EDM will provide adequate results.

10-4. Local Coordinate Systems for Reference Network Surveys

The specifications outlined in this chapter refer to a 3-D local coordinate system where x and y are the horizontal coordinates and *z* is the vertical (i.e., height) coordinate. The X-Y grid system used is usually a local coordinate system or construction system. Connections to geographic systems or SPCS are not required or needed for PICES deformation surveys. The *z* coordinate can either be: local orthometric height if conventional survey or conventional combined with photogrammetric survey observations are used; local Euclidean height if photogrammetric survey observations are used only; or the local ellipsoidal height if only GPS, GPS plus conventional, and/or photogrammetric survey observations are used.

10-5. First-Order 3-D Deformation Monitoring Specifications

The following sections detail the minimum specifications that should be followed to achieve the reference network accuracies detailed in Table 10-1.

10-6. Network Design

When designing a network for deformation monitoring, it is important to keep in mind that: a structure and its foundations form a whole that is embedded in the surrounding terrain which may also impact the observations; abnormal structural behavior may occur either quite rapidly or gradually over time; and if abnormal deformation does occur, analysis of deformation measurement data can help to identify its cause. Therefore, a network design should be capable of monitoring both short- and long-term influences, as well as be capable of distinguishing between movement of the structure, its foundation, and surrounding terrain. Analysis of behavior for the short term consists of collection and analysis of specific data frequently, while long-term behavioral analysis consists of collecting and examining more differentiated data. Instrumentation and procedures for short-term analysis must be easy to operate and follow, providing measurements that are relatively easy to interpret. Instrumentation and procedures for long-term analysis are conducted less often, therefore instrumentation and procedures are typically more sophisticated, often requiring the aid of specialists in order to be done properly. The specific design of the deformation network can be used to determine the network configuration, instrumentation, and accompanying procedures to be followed.

a. There are no existing PC-based software packages specifically intended for deformation network design. Even though this is the case, network adjustment software using rigorous adjustment methods (e.g., method-of-least-squares-based adjustment software) can be used to carry out deformation network design through preanalysis of alternative designs. Deformation network design is then facilitated through a “sensitivity analysis” in which prechosen movements are tested to determine if they are detectable with the particular design alternative. Using network adjustment software in such a manner should be left to those with experience in such and should not be done by the novice.

b. Recognizing the complexity and difficulties of using network adjustment software for deformation network design, the following empirical relationship was developed and will be used to determine the detectable single point movement possible in a particular deformation network design:

$$S_p = P_p * \sqrt{2} \tag{10-1}$$

where

S_p = detectable single point movement

P_p = precision of the point position

P_p can be further defined based on the definition of the points position. For a 1-D (i.e., *z*) point position definition:

$$P_p = S_z \tag{10-2}$$

For a 2-D (i.e., *x* and *y*) point position definition:

$$P_p = \sqrt{(S_x^2 + S_x^2)} \tag{10-3}$$

For a 3-D (i.e., *x*, *y*, and *z*) point position definition:

$$P_p = \sqrt{(S_x^2 + S_y^2 + S_z^2)} \tag{10-4}$$

where

(for Equations 10-1, -2, and -3):

S_x, S_y, S_z = estimated precision, at the 95 percent confidence level of the respective *x*, *y*, and *z* coordinates

c. In lieu of network adjustment software that use a rigorous method of adjustment, Equations 10-1 through 10-4 will be used as the basis for deformation network design. The explanation for the use of a detectable single point movement for deformation network design should be obvious: a network is less sensitive to the movement of one point (i.e., it is more sensitive to the movement of a group of points); therefore, the magnitude of a single point is used to define the criteria for the deformation network design for deformation surveys.

d. In a network observed by conventional survey methods, there can be weak areas with respect to the precision, reliability, and detectability of movements because of the lack of redundant measurements. Subsequently, the network design for a network observed by conventional survey methods is very critical. By contrast, a network observed by photogrammetry, GPS, or a combination thereof typically provides a large number of redundant measurements and its network design is not as critical. Regardless of this phenomenon, the empirical relationships in Equations 10-1 through 10-4 are valid for deformation network design, exclusive of the method used to monitor it, and will be used when practicable.

10-7. Monumentation

A monument used for deformation monitoring is any structure or device which serves to define a point in the deformation survey network. It must have long-term stability with respect to the area surrounding it of less than 0.5 mm both horizontally and vertically. A monument can be classified as either a reference point or an object point. A reference point typically is not located on the deformation structure and is to be “occupied” during the deformation survey. An object point is located on the deforming structure and is to be “monitored” during the deformation survey. The monumentation described in the following paragraphs will be used for both horizontal and vertical observations as these observations are taken during a typical deformation survey. Therefore, the use of deeply set monuments used for either horizontal or vertical control surveys alone typically are not used in a deformation network design and subsequent observations.

10-8. Reference Point Monuments

a. Reference points installed in the earth shall be installed so as to have a depth equal to at least twice the depth of frost penetration in the project area. These reference points can be either a steel pipe pile or cast-in-place reinforced concrete pile (Figure 10-1 a and b). If a steel pipe pile is used, the nominal diameter will be no less than 20 cm, while the wall thickness will be no less than that for standard weight pipe. If a cast-in-place reinforced concrete pile is used, the nominal diameter will also be no less than 20 cm.

b. Installation of the reference point will be as follows:

(1) Steel pipe pile. A steel pipe pile will be installed by driving it until refusal. If refusal occurs at a depth of less than twice the depth of frost penetration in the project area, the pile will be removed and its

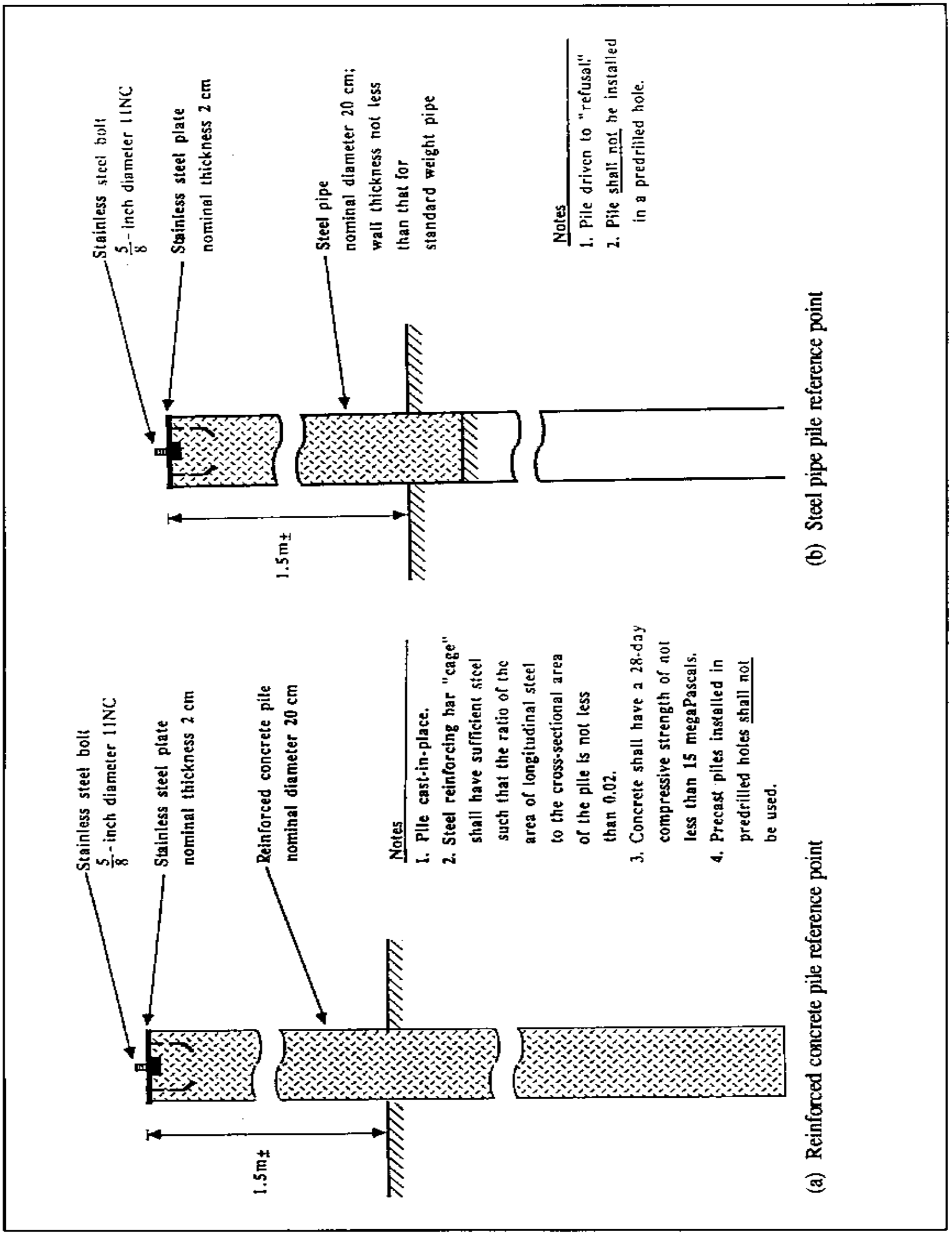

Figure 10-1. Reference points

installation attempted in another location. Steel pipes placed in oversized predrilled holes and backfilled will not be used as reference points.

(2) Cast-in-place reinforced concrete pile. A cast-in-place reinforced concrete pile will be installed by first drilling a hole to at least twice the depth of frost penetration in the project area. The cage of steel reinforcing bars used will have a cross-sectional area of steel to concrete of not less than 0.02. After the cage is formed, it is placed in the hole. Concrete with a 28-day compressive strength of not less than 15 megaPascals is then poured into the form. Precast reinforced concrete piles driven into predrilled holes or placed in oversized predrilled holes and backfilled will not be used for reference points.

c. If the length of line of observation is less 1 km, it is preferable for the reference point to extend above ground level to a convenient height (e.g., 1.5 m) where the equipment can be force centered. Typically, at the top of such a reference point pile, a stainless steel plate not less than 2 cm thick is cast into the top of the pile using a minimum of four steel reinforcing bar anchors welded to the underside of the plate. In the center of the plate, a 5/8-inch-diameter 11NC steel bolt is welded to the plate to allow for survey equipment to be attached.

d. Where cold weather conditions dictate, an insulation sleeve may need to be installed around the reference point pile that extends above the ground. The installation of a sleeve is to eliminate the possibility of temperature-induced pile movements that may be the result of solar radiation (i.e., temperature variation due to time of day). When this is the case, the sleeve should have an R value of not less than 10.

e. For pipe piles terminating at or slightly below ground level, a convex stainless steel plate and stub will be installed as described above. The plate will be convex as required for leveling observations and will have an etched cross at the highest point of the convex surface for horizontal observation. It is recommended that such piles also have a cylindrical rim and cover around it for protection. If a cylindrical rim and cover is used, it is further recommended the cover be buried so as to be easily recoverable with a metal detector, as well as to minimize the chance of vandalism.

f. If possible, the reference points should be installed at least a year prior to their use to minimize the effects of pile rebound and shrinkage. If this is not practicable, no less than a month prior to their use will suffice.

g. Reference points installed in rock or concrete shall consist of a stainless steel plate as described above, except with a steel reenforcing bar stub welded to the underside. For installation, a hole at least 50 percent larger than the stub will be drilled into sound rock or concrete. The plate with the stub attached will be secured to the rock or concrete using adequate epoxy adhesive to completely fill the void between the stub and the rock or concrete.

10-9. Object/Target Point Marks

a. Object points installed in the earth will consist of a nominal 3-m length of square steel hollow structural section with a nominal side length of 5 cm and a wall thickness not less than that for a standard weight square steel hollow structural section (Figure 10-2). The base of the section is sharpened by cutting it at a 45-degree angle. Welded approximately 15 cm from the base is one length of 10-mm-thick 20-cm-diameter circular helix with a pitch of 7 cm. Welded to the top of the pipe is a steel plate not less than 5 mm thick. In the center of the plate a 5/8-inch-11NC steel bolt onto which survey equipment is to be connected is drilled through and welded to the plate. If practicable, some method (e.g., through the use of a cap) should be used to protect the threads of the bolt during the time survey equipment is not attached.

b. Object points set directly in rock or concrete may be either a steel bolt onto or a steel insert into which survey equipment is force centered. The installation of these types of object points will be as follows:

(1) Steel bolt. The steel bolt will be drilled through and welded to a 5-cm-diameter, 1-cm-thick steel plate. A steel reinforcing bar stub of suitable length will be welded to the head of the bolt. A hole approximately 50 percent larger than the stub will be drilled in sound rock or concrete. The plate with the stub attached will be secured to the rock or concrete using adequate epoxy adhesive to completely fill the void between the stub and the rock or concrete. Once again, if practicable, some method (e.g., through the use of a cap) should be used to protect the threads of the bolt during the time survey equipment is not attached.

(2) Steel insert. Steel inserts have been designed by various manufacturers as off-the-shelf items. Manufacturer instructions for proper installation of the insert should be followed.

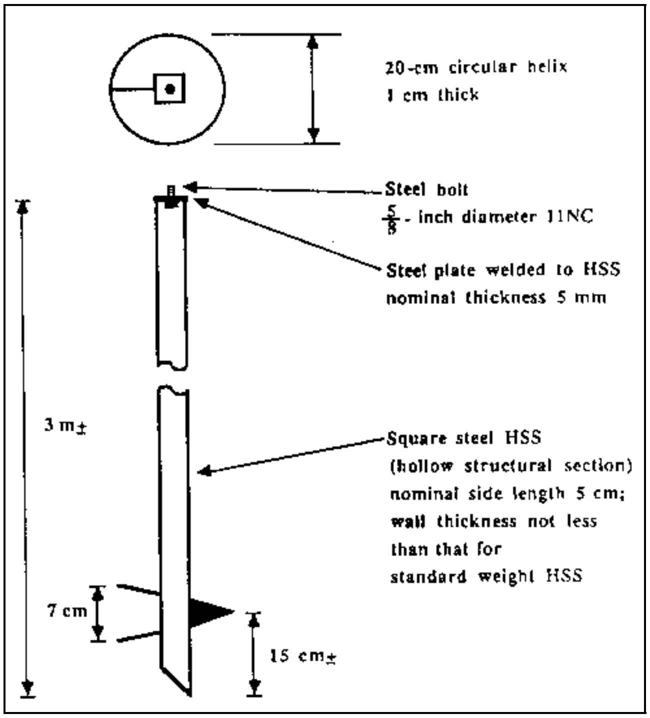

Figure 10-2. Hollow steel structure with helix base object point

c. Object points on materials (e.g., steel, masonry, etc.) other than described in the previous paragraphs will be permanently affixed. For object points to be mounted on steel, a steel bolt welded to the steel may be suitable. For masonry, or other material, a steel bolt, plate, and rear stub or a steel insert may be suitable.

d. For all reference and object points installed, an applicable identifier (e.g., numeric or alphanumeric) will be stamped on the point as appropriate. A permanent record will be kept of the identifier, description, location, and condition of each reference and object point.

e. Further information on specific monument design and installation is provided in EM 1110-1-1002.

10-10. PICES Targets

A target used for deformation monitoring is a device with a well-defined aiming point which is placed vertically over or attached to a monument. The purpose of a target is to permit making measurements to the point over which it is installed when the deformation survey network is observed. Such a device typically is installed only for the period of the survey. In some cases, the monument may be a target itself.

a. Targets used for angular measurement, either horizontal, vertical, or both, will be either:

- Standard force-centered target sets designed for one-second theodolites;
- Standard force-centered target set/prism combination used with a particular total station or theodolite and EDM;
- A target made of the material from which typical force-centered prisms are made;
- The monuments themselves.

b. Target set/prism combinations not matched to a particular total station will not be used. Also, target set/prism combinations for total stations which are noncoaxial will be tilting target set/prism combinations so the target set/prism can be tilted together to allow for alignment with the line of observation.

c. Targets for taped distances will be the monuments themselves.

d. Targets for EDMs will be the prisms included with the EDM. Prisms not matched to a particular EDM will not be used.

e. Targets for spirit-leveled height difference measurements will be the monuments themselves. If the monuments are steel inserts, the targets will be stainless steel plugs designed for the purpose. If more than one plug is to be used on a project, the plugs will be of the same size.

f. The preferred targets for photogrammetric-based deformation surveys typically consist of a white dot on a black background. The diameter of the white dot is chosen so as to yield an average image diameter of 60 microns. The black background typically is five times the diameter of the white target.

g. The targets for GPS-based deformation surveys generally are the monuments themselves.

10-11. PICES Instrumentation

The instruments typically used for deformation surveys are described in the following paragraphs.

a. If using an optical theodolite, it shall have a telescope magnification of 30 times or better, a plate level with a sensitivity of 20 seconds per 2-mm graduation or better, an automatic vertical circle compensator, and a coincident micrometer direct to 1 second or better.

b. If using an electronic theodolite, it will have the same characteristics of an optical theodolite or better, but the circle reading system will be accurate to 3 seconds or less.

c. Distances of 10 m or less can be measured with a steel or invar tape. An EDM will not be used to measure distances less than 10 m. Distances of 30 m or less can be measured with a tensioned steel tape, invar tape, or invar wire which can be attached to the steel bolt or insert directly, a subtense bar, or an EDM. An EDM is the preferred instrument for distances beyond 30 m. Microwave-based EDM systems will not be used. It is not necessary to measure any distances with multiple wavelength-based instruments.

d. Barometers will be capable of 2-mm mercury precision or better. Thermometers will be of 1 degree Celsius readings or better.

e. If spirit leveling is done, the instrument will be an automatic level with telescope magnification of 40 times or better, compensator with a sensitivity of 10-second per 2-mm level vial graduation, parallel plate micrometer capable of 0.1-mm readings. The compensator will be a free suspension one and not a mechanical one so as to minimize the effect of possible electromagnetic fields. The rod to be used should be an invar, double scale rod or one with a permanently attached circular level attached - both having graduations equal to the range of the parallel plate micrometer.

f. All equipment will have an optical plummet either incorporated in it or be capable of being used with a detachable tribrach that has an optical plummet.

g. When performing photogrammetric-based deformation surveys, only metric cameras will be used. Typically, only one camera is necessary as it is moved from station to station. The instrument used for image coordinate measurement (e.g., monocomparator, stereocomparator, or analytical stereocomparator) will be capable of 1-micron or better resolution.

h. When performing GPS-based deformation surveys, the receiver used will be at least geodetic quality, multichannel, single frequency, and capable of at least one-minute data sampling. At the least, the receiver should be capable of recording the GPS carrier frequency, receiver clock time, and signal strength for each data sample. A receiver is required for each reference and object point. The same receiver/antenna combination should be used for each setup.

10-12. Equipment Adjustment and Calibration

All equipment used for deformation monitoring surveys will be maintained in adjustment and calibration between use so as to minimize possible errors that may result from the equipment being out of adjustment. Manufacturer specifications will be used as the basis for the adjustments and calibrations.

a. No adjustments need be made to barometers, thermometers, and EDMs. Instead, systematic instrument errors will be determined by calibration and standardization procedures and the associated observations made with the equipment will be corrected accordingly.

b. If a tribrach is used, the only adjustment and calibration necessary is to the optical plummet if so equipped.

c. A metric camera will not be adjusted or calibrated during a survey session. If adjustment or calibration need be done, it will be done only by personnel qualified in such. The same applies for the image-coordinate measurement devices. Such activities should be done as a matter of habit at least once a year or more if necessary.

d. GPS equipment typically does not require adjustment and calibration. If a piece of GPS equipment does not appear to be operating correctly, the manufacturer should be consulted.

10-13. Survey Procedures

Formalizing a particular procedure to be followed that meets all circumstances is difficult, but generalized procedures can be developed and are detailed in the following paragraphs.

a. Angle measurement with a theodolite. When using a theodolite for angle measurement, it will be accurately plumbed over the occupied point by either attaching the theodolite to the point with a tribrach or using a tripod and tribrach with an optical plummet, as applicable. Once measurements are made, the level of the instrument will be checked. If found to be greater than 10 seconds, the measurements will be done again with a leveled instrument. When an electronic theodolite is used, the level of the instrument will be kept to less than 2 minutes.

(1) The reticule should be focused first and then the objective lens of the theodolite.

(2) If possible, observation should be limited to days when the weather conditions are fairly neutral (e.g., cloudy day with a light breeze). Days with temperature extremes should be avoided. If an instrument must be used when the temperature is hot, it should be protected from the sun by an umbrella.

(3) Observation of zenith angles, where necessary, should be limited to between 10 in the morning and 3 in the afternoon in order to minimize the chance for vertical refraction. When elevation differences are determined by zenith angles with a theodolite, such equipment will not be operated near high electromagnetic producing devices (e.g., transformers, high voltage power lines, etc.).

(4) Directions and zenith angles will be observed in four rounds. In all theodolites, face left and face right point and reads will be made on all targets. The exception is a few types of theodolites that have auto calibration based on point and reads to one target. All horizontal and vertical circle readings will be recorded manually or electronically to 0.1 second.

b. Distance measurement with a tape. Distances measured between monuments will be made point to point whenever possible. When using a steel or wire invar tape, the uncorrected distance to the millimeter, temperature, tension applied, slope, unsupported length(s), and the standardization error of the tape will be recorded. If unable to measure point to point, a tripod and theodolite will be plumbed and leveled over the points and the distance measured between the trunnion axis of the setup. If tensioned equipment is used, in addition to the previous measurements to be made, the uncorrected distance should be measured to 0.01 mm. Distance measurements made by tape will be independently made at least two times by repeating the setup required to make the measurements.

c. Distance measurement with a subtense bar. If measuring the distance with a subtense bar, the subtense bar and theodolite will be plumbed and leveled over the points defining each end of the line of observation as described in the previous paragraphs. The optical sight will be used to set the subtense bar perpendicular to the line of observation. The angle subtended by the subtense bar will be measured reiteratively four times by the theodolite. Do not forget to record the height of the instrument (i.e., HI) and height of the target (i.e., HT) to at least 0.5 cm for later reduction of the distance to point to point.

d. Distance measurement with an EDM. If measuring the distance with an EDM (including those incorporated within total stations), the EDM will be accurately plumbed and leveled over the point, or force-centered. Observations with an EDM will not be conducted over terrain where extremes may be present (e.g., across a valley or river) that under adverse weather conditions may produce large errors related to atmospheric conditions and temperature along the lines of observations. To this end, observations with an EDM also should be limited to days when favorable atmospheric conditions (e.g., slightly cloudy with a light breeze) are prevalent. An EDM should never be operated near external electromagnetic field producing sources.

(1) Prior to its use, an EDM should be allowed to "warm up" according to manufacturer specifications. Also, the EDM should be operated in the manufacturer recommended range of ambient temperatures (e.g., typically -20 and +40 °C) and with fully charged batteries.

(2) Prior to measurements with the EDM, the target prism will be set perpendicular to within 10 degrees of the line of observation. When actual measurements are made, the prism will be adjusted to maximize the strength of the signal.

(3) Each EDM measurement between object and target station will be done iteratively at least four times by resetting the instrument and performing the observation again.

e. Pressure and temperature measurement. Pressure and temperature will always be measured at the instrument stations and at the target station if it is a reference network point (rather than an object point). (Note that the RLR/Robertson Method is independent of pressure/temperature measurements.) Temperature and pressure will always be measured in a location shaded from the sun, exposed to any wind, and at least 5 feet above the ground and away from the observer and instrument. Corrections to temperature and pressure will never be applied when the observations are made, but will be applied when the observations are reduced.

f. Additional observation requirements. When observations are made, in addition to the other standard information required for a survey, the following should be recorded in the field book:

(1) Pressure to 2 mm.

(2) Temperature to nearest 1 °C.

(3) HI of ED and prism to 0.5 cm, HI of theodolite and target to 0.5 cm if distance is measured, 0.5 mm if both distance and height differences are measured.

g. Leveling operations. When determining elevation by precise spirit leveling, the following guidelines will be followed:

(1) Either one or two double-scale invar rods will be used. For short runs, traditional three-wire procedures are allowable.

(2) Level lines will be run in only one direction.

(3) Either observed stadia or a cloth tape may be used to balance the foresight and backsight distances between two deformation survey network points. If the distances cannot be balanced, they will be recorded so that the height difference can be adjusted during data reduction.

(4) If using one level rod, it will be moved from backsight to foresight as quickly as possible and readings made. The readings will be recorded manually in the field book or electronically to 0.01 mm.

(5) The maximum length of the line of sight should not be more than 150 feet.

(6) The line of sight will not be less than 1.5 feet above the ground.

(7) Leveling operations will be conducted under the recommended favorable conditions cited for previous instruments. Also, automatically leveled equipment will not be operated near electromagnetic field generating sources as recommended in previous paragraphs for other instruments. HI of the theodolite and target will be recorded as detailed herein.

(8) Trigonometric leveling as detailed in the paragraphs on EDM operation can be used to determine height differences in lieu of spirit leveling.

(9) The importance of measuring the HI to the accuracies cited cannot be overemphasized. A blunder in the measurement of the HI will transfer through subsequent measurements, reductions, and adjustment, giving inaccurate results. In an effort to measure the HI correctly, proper targets and HI measuring instruments, as well as sound HI measurement procedures, should be followed at all times.

(10) When performing leveling operations, the coefficient of refraction determined for data and applicable for the observation period will be applied.

h. Photogrammetry operations. When performing photogrammetric-based deformation surveys, the metric camera used will be mounted in or on a suitable camera platform (e.g., camera tripod). During exposure, movement of the camera will be minimized. If using an airplane or helicopter for the platform, a camera with an image motion compensator must be used. Typically, 5 to 20 exposure stations are necessary to ensure sufficient precision for the object point coordinates are determined. To ensure the whole photo-taking portion of the survey is performed correctly, it is highly recommended that only experienced personnel be used for this phase of the survey. The photogrammetric reduction process also should be done by experienced personnel trained in image coordinate measurement with the appropriate equipment. If practicable, it is recommended that this process be automated in order to eliminate the possible gross errors possible with self-calibration. EM 1110-1-1000 should be referred to for more specifics on the photogrammetric process.

i. GPS operations. When performing GPS-based deformation surveys, the procedures will be done in accordance with EM 1110-1-1003.

j. Preprocessing of data. Preprocessing of data is done to minimize the amount of subsequent calculations required and eliminate erroneous data.

k. Preprocessing conventional survey data. Preprocessing of conventional survey data consists of applying a rejection test at the time the observations are made in order to reject probable outliers and atmospheric, instrumental, standardization, and geometric corrections so data can be imported directly into subsequent adjustment software. Gravity corrections typically do not need to be applied due to the small areal extent of the data collection. Preprocessing of conventional survey observations can either be done manually or by appropriate verified and validated PC based programs.

l. Preprocessing photogrammetric-based survey data. Preprocessing of photogrammetric-based survey data will include the screening of measured image coordinates in order to reject observations which are outliers and to determine 3-D object coordinates and associated variance-covariance matrix in the local coordinate system. Determination of the 3-D object coordinates should be accomplished by a computer-based bundle adjustment program with self-calibration. Also, in the bundle adjustment, the focal length, horizontal position of the principal point, and coefficients of radial lens, asymmetric lens, and photographic lens distortion and photographic media unflatness will be treated as weighted unknowns. If the distance between exposure is kept to what is recommended, atmospheric refraction also will be neglected. If using GPS observations in conjunction with the photogrammetric process, the appropriate earth curvature correction will need to be applied.

m. Preprocessing GPS survey data. Preprocessing of GPS survey data at a minimum will include determination of the 3-D coordinate differences and associated variance-covariance matrix in the local 3-D coordinate system used for all baseline observed and screening of these reduced vectors to eliminate possible outliers. Table 10-2 lists some rejection criteria for preprocessing deformation survey data, as well as action that can be taken if the data do not pass the rejection criteria.

(1) When a mean uncorrected distance is determined using a steel or invar tape or steel tape, invar tape, or invar wire measuring unit, the following corrections will be applied to determine true distance:

- Temperature correction between the observed and standardized tape distance (ignore if using an invar tape).
- Tension correction between the observed and standardized tape distance.
- Correction due to the unsupported length(s) of the tape.
- Slope correction if applicable.
- Correction due to standardization error.

(2) If the distances collected with an EDM are not rejected, a single uncorrected distance will be computed as the mean of the four independent measured distances. When the mean uncorrected distance is determined, the following corrections will be applied to determine true distance:

- Atmospheric corrections based on the atmospheric conditions during observation and standard atmospheric conditions.
- Correction due to the additive constant for the particular EDM and prism combination.
- Correction due to the standardization error of the EDM.
- Geometric corrections, to include correction due to unequal HI for EDM, prism, theodolite, and target, arc distance to slope distance corrections, slope distance to horizontal distance correction, and horizontal distance at station elevation to horizontal distance at reference elevation correction.

(3) If data collected with an automatic level are not rejected, a single height difference will be computed as the mean of the height difference computed and the height difference computed from the right scale readings. If the foresight and backsight readings are unbalanced, the single height difference will be corrected for vertical collimation error.

n. Analysis of collected data. Prior to analysis of collected data, the adjustment and localization of deformation survey data typically are separated into the 2-D (i.e., horizontal) and 1-D (i.e., vertical) components for analysis. More frequently, though, analysis is being done on 3-D vectors, especially when GPS-based deformation surveys are being used.

(1) The analysis of deformation survey data will include:

- Adjustment of observations made in each epoch in order to determine point coordinates, associated variance-covariance matrices, and other information required for localization of the deformation.
- Localization of deformations between epochs in order to determine statistically significant point movements.

(2) The first step in analysis of the collected data is the survey adjustment. For conventional survey observations, the standard deviations listed in Table 10-3 will be used in the survey data adjustment.

(3) For adjustments where conventional survey data and/or GPS survey observations are combined with photogrammetric survey observations, the localized 3-D coordinates (i.e., x, y, z coordinates) and associated variance-covariance matrix from the photogrammetric survey

Table 10-2
Rejection Criteria For Preprocessing of Deformation Survey Data

Type of Instrument	Type of Measurement	Test	Action to Follow if Data are Rejected
Theodolite[1] or Subtense Bar[1] or Theodolite[1] (Trigonometric Leveling)	Angle Angle Elevation	1. Reduced data must be less than 2 seconds from the mean reduced direction --> Otherwise, reject 2. Reduced zenith angle not being used to compute a height difference must be less than 4 seconds from the mean reduced direction --> Otherwise, reject 3. Reduced and corrected zenith angle not being used to compute a height difference must be less than 2 seconds from the mean reduced and corrected zenith angle --> Otherwise, reject	Re-observe the portion of the survey rejected
Steel or Invar Tape	Distance	1. Difference between two independently measured distances must be less than 2 mm --> Otherwise, reject	Remeasure the distance rejected
Steel Tape, Invar Tape, or Invar Wire Measuring Unit	Distance	1. Difference between two independently measured distances must be less than 0.02 mm --> Otherwise, reject	Remeasure the distance rejected
EDM Distance or EDM Elevation (Trigonometric Leveling)		1. Maximum difference among the four independent measured distances must be less than 5 mm --> Otherwise, reject	Remeasure the distance rejected
Automatic Level Setup	Elevation	1. Difference between readings on the left- and right-hand scale must be within 0.25 mm of rod constant --> Otherwise, reject 2. Difference between height difference determined from the foresight and backsight readings on the left rod scale and that determined from foresight and backsight readings from the right scale must be less than 0.25 mm --> Otherwise, reject	Re-observe the portion of the survey rejected
Network of Level Setups	Elevation	1. Height difference misclosure in a loop must be less than 3 mm * $\sqrt{(K \text{ in km})}$ --> (Minimum = 1 mm) Otherwise, reject	Formulate different loops to determine height differences between points common to loops which have been rejected; or re-observe the portion of the survey rejected
Level and Meter Rule	HI	1. Difference between two independent readings must be less than 0.5 mm --> Otherwise, reject	Remeasure the distance rejected
Mono-comparator, stereo-comparator, or stereo comparator	Photo image coordinates	1. As applied by photogrammetry software for hardware used --> Otherwise, reject 2. Discrepancy between double measured image coordinates is less than 2 microns --> Otherwise, reject	Remeasure image coordinates
GPS Receivers	Horizontal coordinates and elevation	1. Tests as detailed in EM 1110-1-1003	Re-occupy baseline

[1] When performing these data reductions, no atmospheric, instrumental, standardization, and geometric corrections are necessary for angular observation made with a theodolite, except in the case of zenith angles which are observed for the purpose of determining height differences (in which case, earth curvature and refraction need be considered). Because a deformation survey is on a localized network, skew-normal, arc-to-chord, and normal section to geodetic correction need not be applied.

Table 10-3
Standard Deviations to be Used in Deformation Survey Data Adjustment

Measurement	Standard Deviation (σ)[1]
Direction	2 arc seconds
Zenith Angles - when simultaneous or near simultaneous zenith angles are observed and corrected for earth curvature and computed coefficient of refraction	in arc seconds, $\sqrt{(4 + ((61{,}879.5 * S)/(2 * R))^2)}$
Zenith Angles - when 1-way zenith angles are observed and corrected for earth curvature and computed coefficient of refraction	in arc seconds, $\sqrt{(4 + ((123{,}759.0 * S)/(2 * R))^2)}$
Zenith Angles - when 1-way zenith angles are observed and corrected for earth curvature and estimated coefficient of refraction	in arc seconds, $\sqrt{(4 + ((206{,}265 * S)/(2 * R))^2)}$
Distance measured with a steel or invar tape	2 mm
Distance measured with a steel tape, invar tape, or invar wire measuring unit	0.02 mm
Distance measured with a subtense bar	in mm, $\sqrt{(2.25 + (5 * S^2 * 10^{-3})^2)}$
Distance measured with an EDM	in mm, $\sqrt{(25 + (2 * S_s * 10^{-3})^2)}$

[1] σ = estimated standard deviation
S = approximate horizontal distance in km
R = approximate radius of the earth = 6,370 km
S_s = approximate spatial distance in meters

observations and subsequent bundle adjustment and self-calibration will be included. When only using photogrammetric survey observations, the free network constraints (i.e., inner constraints) will be used to define the datum. When photogrammetric survey observations are combined only with conventional survey observation, the datum will be defined by the constraints used in a conventional survey adjustment. When photogrammetric survey observations are combined only with GPS survey observations, the GPS survey observations will be used to define the datum (e.g., location, orientation, scale).

(4) When GPS survey observations are adjusted, they will be adjusted using either the GPS-based localized 3-D coordinates or coordinate differences and associated variance-covariance matrices in accordance with EM 1110-1-1003.

(5) A rigorous form of adjustment (e.g., method of least squares) will be used in the network adjustment software for reducing the deformation survey data. It will be capable of computing the following: the point coordinates and associated variance-covariance matrix, point error ellipses, standardized residuals for each observation, quadratic form of the residuals, total redundancy of the network, and the estimated global variance factor. If only conventional survey observations are to be adjusted in a network, the following additional information is required to be computed: redundancy number of each observation, quadratic form of residuals for each observation type (i.e., directions, zenith angles, etc.), sum of redundancy numbers for each observation type, and the estimated variance factor for each observation type.

(6) The adjustment will take place in two basic steps. The first step is the adjustment of observations and localization of deformations in the reference network. The second is adjustment of observations and localization of deformations in the entire network.

(7) In the first step, all network reference points and all observations directly connecting reference network points will be included in the adjustment. Table 10-4 details the ordinary minimum constraints to be used in the adjustment of conventional survey observations.

If standardized residuals are computed, the observations having values greater than 4 should be examined as possible blunders. Observations identified as blunders will be deleted from the adjustment and the adjustment redone. The estimated global variance factor and estimated

Table 10-4
Minimum Constraints to be Used in the Adjustment of Conventional Survey Observations[1]

Network Type	Minimum Constraint
1-D (i.e., *z*)	*z* of 1 point held fixed
2-D (i.e., *x*, *y*) with distance	*x* and *y* of 1 point held fixed azimuth of 2nd point held fixed (standard deviation of azimuth = 0.1")
2-D (i.e., *x*, *y*) without distance	*x* and *y* of 2 points held fixed
3-D (i.e., *x*, *y*, *z*) with distance	*x*, *y*, and *z* of 1 point held fixed azimuth and zenith angle to 2nd point held fixed zenith angle to 3rd point held fixed (standard deviation of azimuth and zenith angles = 0.1")
3-D (i.e., *x*, *y*, *z*) without distance	*x*, *y*, *z* of 3 points held fixed

[1]*x* = *x* horizontal value
y = *y* horizontal value
z = *z* vertical value (i.e., elevation)

NOTE: 2-D and 3-D network minimum constraints shall be applied to opposite sides of the network.

variance factors for each observation type for the adjustment should be near 1. If they are less than 0.5 or greater than 2.0, all of the input variances should be rescaled by a ratio of the estimated variance factor of observation type divided by the estimated global variance factor and the adjustment rerun. All epochs of observation should then be adjusted in the same manner with regard to the constraints applied, standardized residuals and scale of the input variances.

(8) The results will be examined to ensure the reference points are stable between epochs. Any reference points which are not found to be stable will be excluded from the reference network.

(9) For the second step involving adjustment of observations and localization of deformations in the entire network, the constraints will be either (a) all reference network points found to be stable are held fixed or (b) all reference network points found to be stable forming a free network of points. The first option will result in a very constrained adjustment. Examination of the standardized residuals and rescaling the input variances for each epoch of observation will be done as described in paragraph (7) above.

(10) For localization of deformations in the entire network under step two, the same constraints shall be applied for the epochs being analyzed. The tests detailed herein will be applied. The final results of this step are apparent movements of all object points and any excluded reference points and indications of whether or not these point movements are statistically significant.

(11) The localization and subsequent tests described are being applied to single points to determine their possible movement. Even though this is the case, such localization and testing can be applied to movements of groups of points if a priori information indicating they may have moved is available.

10-14. Final Reports

A final report will be required for each deformation survey or project done where results have been determined. Any deviations from the specifications detailed in this manual will be included in this report.

a. The final report will include figures with plan or cross-sectional views showing the outline of the structure, location of deformation network points and their names. All reference points shown in the figures will be denoted by one symbol, while all object points will be denoted by a different symbol. Point movements will be plotted as vectors with their associated error bars and/or error ellipses. Statistically significant movements will be flagged. Only displacements between two chosen epochs will be plotted on a given figure. Displacement contours will not be plotted.

b. The final report will include a tabular summary of each network adjustment and at a minimum the following information relative to the adjustments made:

- The constraint applied.
- The names of points used.
- The adjusted point coordinates to the nearest 0.1 mm.
- The standard deviations of point coordinates to the nearest 0.1 mm.

- The dimensions of error bars to the nearest 0.1 mm at the one standard deviation level for 1-D network points, dimensions of the axes of the error ellipses to the nearest 0.1 mm at the one standard deviation level plus orientation angle to the nearest 0.1 degree for 2-D network points, and dimensions of the axes of the error ellipses to the nearest 0.1 mm plus out-of-plane angles to the nearest 0.1 degree for 3-D network point coordinates.

- The quadratic form of the residuals.

- The total redundancy of the network.

- The estimated global variance factor.

If the network adjustments use only conventional survey observations, in addition to the above, the following will be required in the final report:

- The redundancy number for each observation.

- The standardized residuals for each observation.

- The quadratic form of residuals for each observation type (e.g., directions, zenith angles, distance, etc.).

- The sum of redundancy numbers for each observation type.

- The estimated variance for each observation type.

c. The final report will include another summary table of each localization of deformations. Such a table will include:

- The constraints applied in the two network adjustments.

- The names of points used.

- The apparent displacements to the nearest 1 mm and associated direction to the nearest 0.1 degree.

- The dimensions of error bars to the nearest 0.1 mm at the one standard deviation level for 1-D network point displacements, dimensions of the axes of the error ellipses to the nearest 0.1 mm at the one standard deviation level plus orientation angle to the nearest 0.1 degree for 2-D network point displacements, and dimensions of the axes of the error ellipses to the nearest 0.1 mm plus out-of-plane angles to the nearest 0.1 degree for 3-D network point displacements.

d. The final report will include figures showing 1-D cumulative displacements of critical points in critical directions versus time. Examples of critical cumulative displacements include movements in the downstream and vertical directions of a small number of points on the crest of a dam or movements in the downhill and vertical directions of a small number of representative points in a earthen dam or levee. The error bar associated with each displacement will be plotted with these displacements. Data from all deformation analyses performed on the project will be included. Statistically significant cumulative displacements will be flagged.